SRR-91-06

Strength Evaluation by Testing of an Old T-Beam Bridge

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Abstract: The evaluation of a highly deteriorated reinforced concrete (RC) bridge by load testing is described. Since the construction drawings are not available, an analytical evaluation cannot be conducted. Simply by relying on engineering judgement, the outer two lanes of the four-lane bridge were closed to the traffic, and the bridge restricted to vehicles of 5 tonnes or lighter. The evaluation by testing showed that the posting load could be safely upgraded to a triple posting of 19, 26 and 35 tonnes and that there was no need to close the lanes to traffic.

Comments:

Key Words: bridge test, strength evaluation, Wyandotte Street Bridge, T-beam bridge, concrete, observed loads, equivalent loads

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1/ Test Summary

Bridge	:	Wyandotte Street Bridge City of Windsor, MTO Site No. 6-3
Type of bridge	:	Reinforced concrete T-beam
Posting	:	5 tonnes and two outer lanes blocked for traffic
Span/width	:	9.8/20.1 m
Angle of skew	:	0°
Date of Test	:	July 18, 1991
Test engineers	:	Baidar Bakht/Aftab Mufti
Job captain	:	Leon Pena
Supervisor, Instrumentation	:	Bill Kwok
Maximum Static Load	:	Two vehicles each weighing about 550 kN and having dual axle groups weighing 354 kN and 336 kN, respectively
Responses measured	:	Vertical deflections and longitudinal strains at the bottom of girders at mid-span and at sides near the supports
Recommendations:	:	Open all the four lanes to traffic and upgrade the posting to either a single posting of 19 tonnes or a triple posting of 19, 26 and 35 tonnes

2/ Bridge Description

The Wyandotte Street Bridge (MTO site No. 6-3) carries an arterial road over an abandoned railway line in the City of Windsor. It is a reinforced concrete T-beam bridge with a clear span of 9.8 m and an overall width of 20.1 m. Records of the date of construction are not available; however, inspection reports indicate that the bridge might have been constructed in either 1912 or 1926.

The construction drawings of the bridge are not available, but enough concrete cover has been lost to indicate that the central four girders comprise built-up steel beams encased in concrete, and that the remaining ten girders are reinforced with bundled steel bars. The cross-section of the bridge showing schematically the fourteen girders carrying the deck slab, is drawn in Figure 1. An overall view of the bridge is shown in Figure 2. It can be seen in this figure that the girders are apparently monolithic with the abutments. The flexural connection between the girders and abutments is also confirmed by exposed reinforcement in an external girder. This exposed reinforcement (Figure 3), clearly indicates the bridge to be of a rigid frame construction.

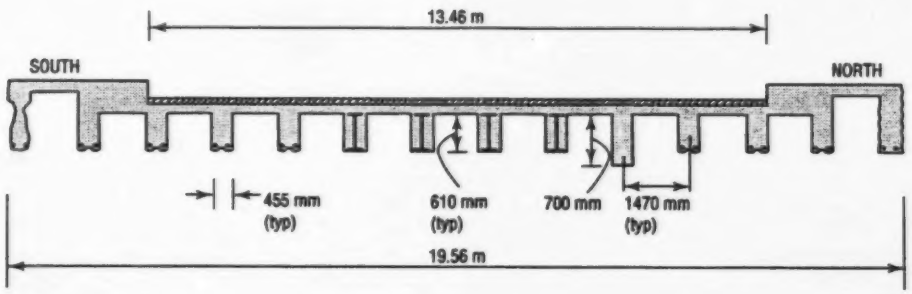


Figure 1/ Cross-section of the bridge at mid-span

Figure 2/ Overall view of the bridge

Figure 3/ A view showing exposed reinforcement provided to ensure flexural connection between the girders and the abutment

At one-third points along the span, the entire width of the bridge is "jacketed" by 460 mm wide steel plates. The plates attached to the bottom of the girders, and the concrete cover to the reinforcement, have been completely lost. However, the plates attached to the sides of the girders and to the underside of the deck slab, are still intact as can be seen in Figure 4. It seems that these steel jackets had something to do with the railway, as they are located under the bridge directly above the middle of each of the two railway tracks, one of which is still in existence.

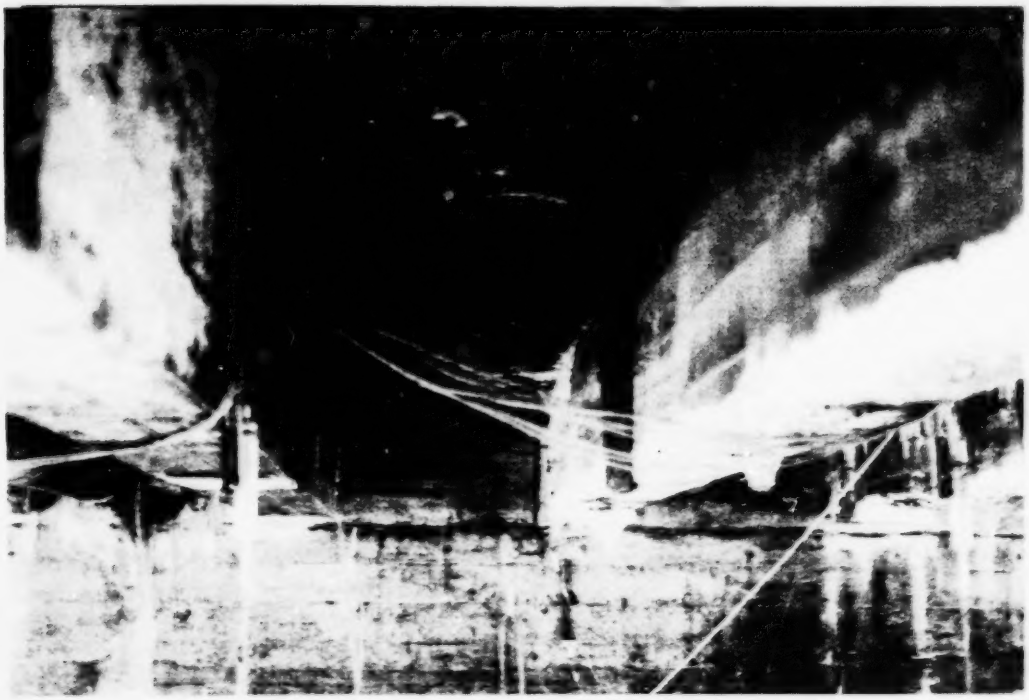


Figure 4/ A view showing steel plate "jacket"

A close-up of the end of one of the middle four girders is shown in Figure 5. This photograph was taken after all the loose concrete was removed just before the test. The exposed rivetted flange of the steel girder can be seen in the photograph along with the apparently sound concrete near the ends. It should be noted that the cover to the steel flange is usually large being of the order of 90 mm.

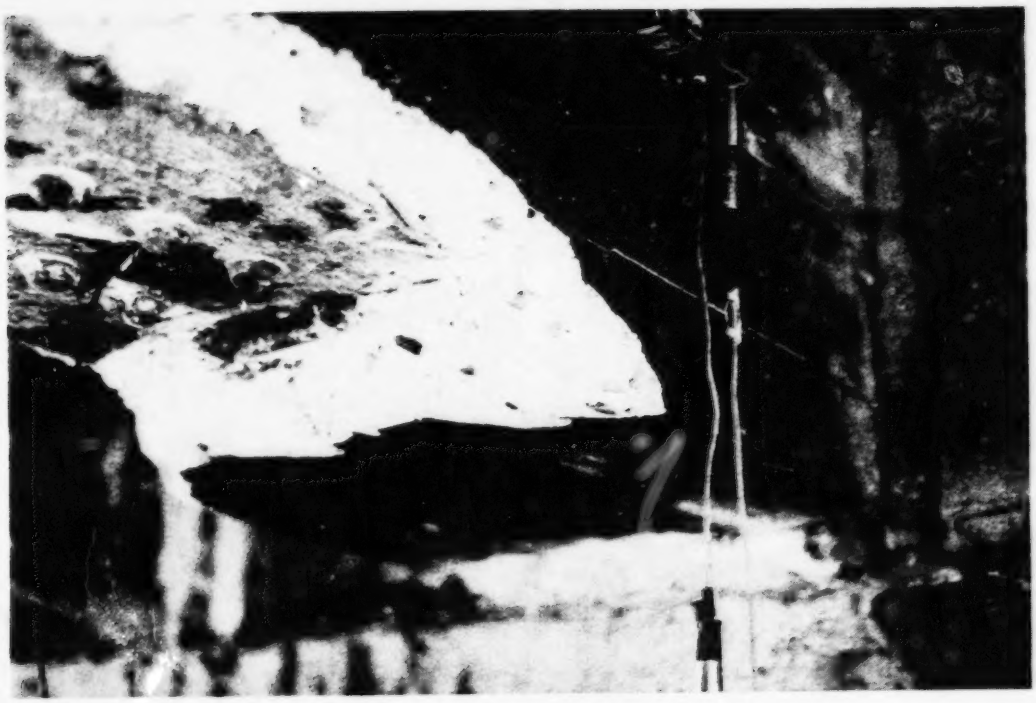


Figure 5/ A view of a girder near a support showing partially exposed rivetted girder

The bundled bar reinforcement for one of the outer beams can be seen in Figure 6. It should be noted that the cover to this reinforcement is entirely lost. A close-up of the lost concrete in the vicinity of the bottom reinforcement and stirrups can be seen in Figure 7.

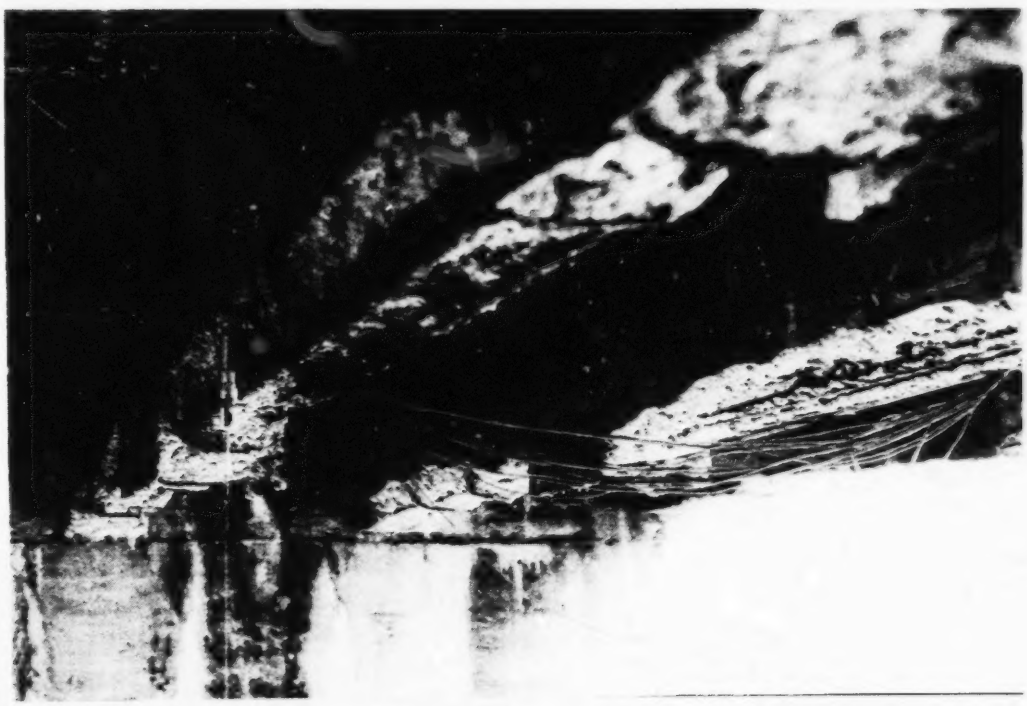


Figure 6/ A view showing exposed bundled bar reinforcement

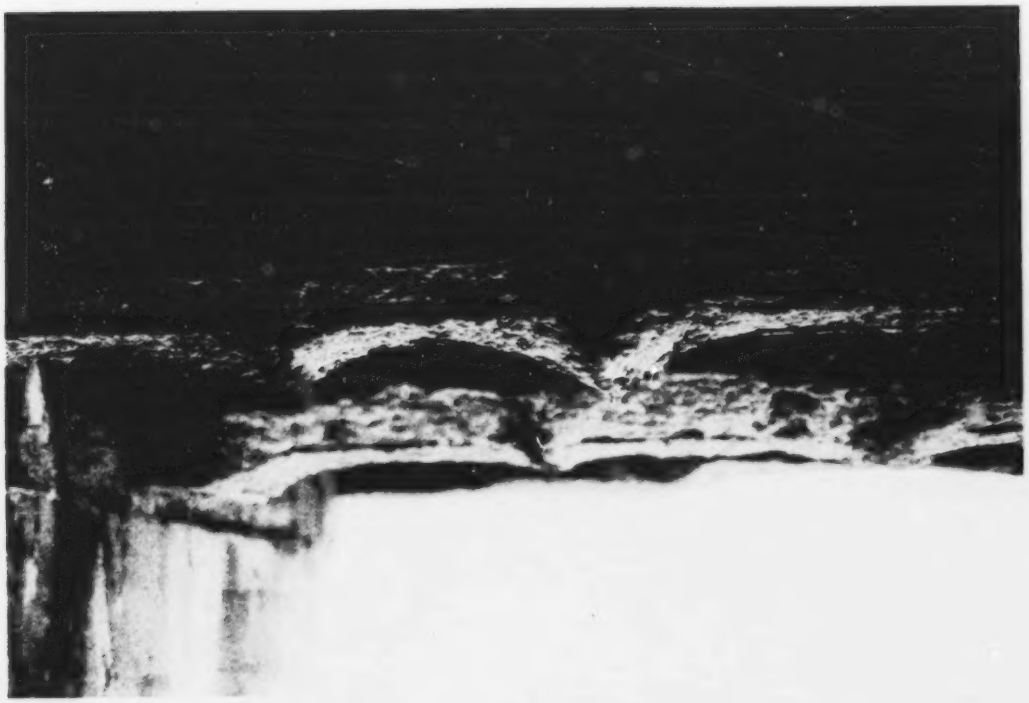


Figure 7/ A close-up of the loss of concrete in the vicinity of reinforcement

The deck slab, blackened presumably by the locomotive emissions, is in good condition except for some exposed bottom reinforcement which can be seen in Figure 8 and also in Figure 4.

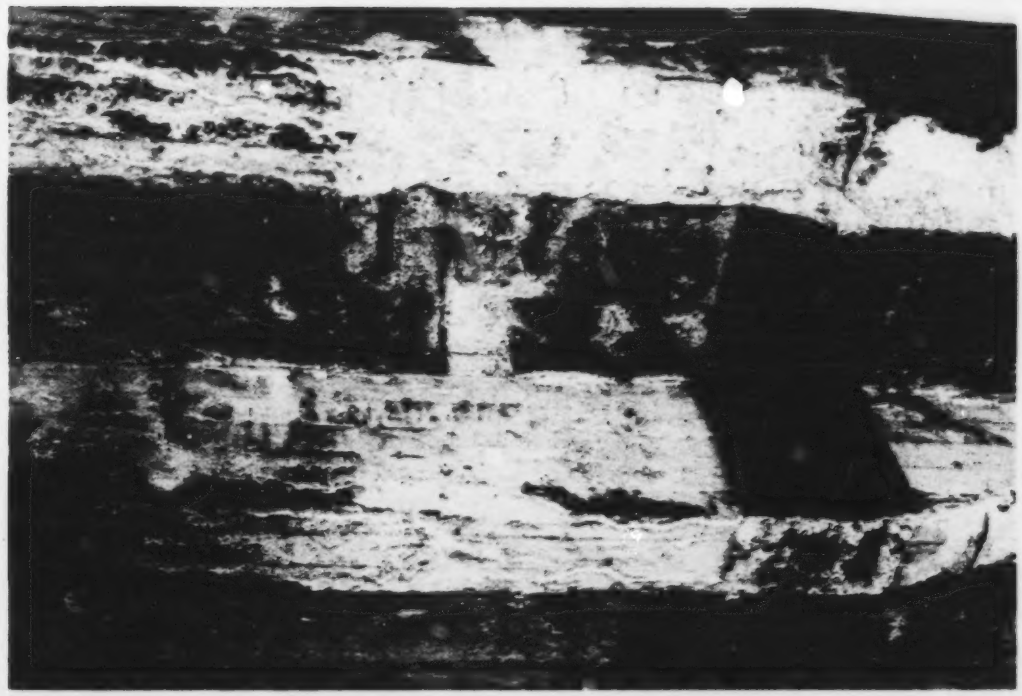


Figure 8/ A view showing partially exposed reinforcement of the deck slab that is generally in good condition

The elevation of a typical interior girder is shown in Figure 9 in which it can be observed that the depth of the girders below the deck slab, in the middle portion, has been reduced to 610 mm. The original depth which remains intact near the ends was about 700 mm.

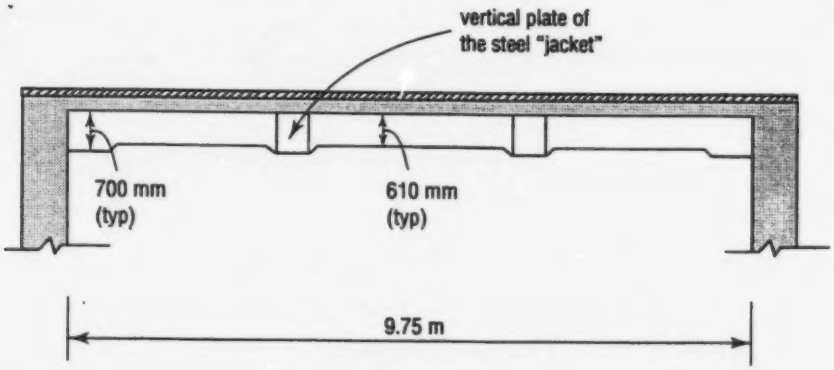


Figure 9/ Longitudinal section through the bridge

As can be seen in Figure 1, the two outer girders on each side of the bridge are slightly deeper than the other girders. The outermost girder on the south side has lost so much concrete that its capacity to carry the sidewalk above it, is questionable. The City of Windsor has closed the southern sidewalk because of the condition of this girder.

Before testing, all the loose concrete form the bridge was removed. It was found that the remaining concrete contained within the reinforcement cage was apparently sound except for the southern outermost girder.

3/ Reasons for Testing

Because of the lack of construction drawings and the observed advanced state of concrete deterioration, the load carrying capacity of the bridge could not be established analytically. A firm of reputable consulting engineers established a posting limit of 5 tonnes by using only engineering judgement. Based, also, on engineering judgement, the two outer lanes of the bridge were blocked and the traffic restricted to the middle two lanes.

The lack of funds prevented the city from replacing the bridge which was declared to be structurally-deficient. Faced with the hardship of having to restrict traffic on one of its arterial roads, the city on the advice of the firm of consultants mentioned above, requested the Ministry of Transportation of Ontario (MTO) to test the bridge. The purpose of the test was to establish a safe but realistic estimate

of the load carrying capacity of the bridge so that the current restrictions could be maintained, upgraded, or even removed.

The test on the Wyandotte Street Bridge was included in the schedule of MTO tests for 1991.

4/ Pretest Analysis

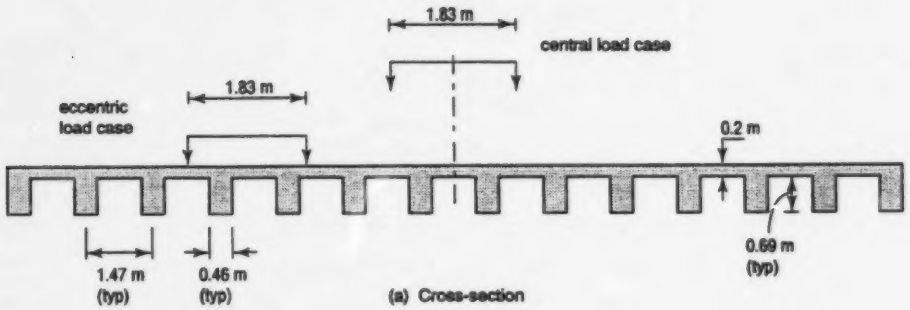
Before loose concrete from the bridge was removed, it was difficult to ascertain, if strain gauges could be applied to the bottom steel of the girders. It seemed that the response of the structure could be monitored only through deflections. A brief pre-test analysis of the bridge was conducted to establish if the transverse distribution pattern of girder moments was substantially similar to the transverse distribution pattern of girder deflections at the mid-span.

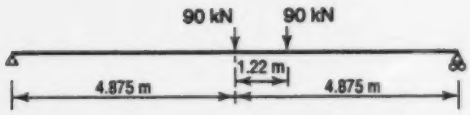
Another reason for the pre-test analysis was the severely deteriorated southern outermost girder which could not be subjected to any test load. The analysis would show if the most eccentric load on the south side of the bridge induces any appreciable load effect on the deteriorated girder, which is separated from the roadway by one girder, as can be seen in Figure 1.

Since the boundary conditions of the girders could not be established with certainty, it was decided to analyze the bridge for two extreme boundary conditions, namely simply-supported and fully-fixed. The semicontinuum analysis program SECANl (Jaeger and Bakht, 1989) was used to analyze the load distribution characteristics of the bridge.

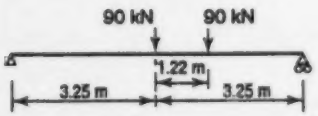
The program SECANl can only analyze simply-supported bridges. However, as shown by Jaeger and Bakht (1989), the semicontinuum method of analysis for simply-supported bridges can also be adopted for the analysis of load effects in the positive moment region of rigid frame bridges; the only adjustment that is required is that the equivalent simply-supported bridge is assumed to have a span L/1.5 where L is the span of the rigid frame bridge.

For the pretest analysis, it was assumed that (a) the span of the bridge is 9.75 m; (b) all girders are of equal stiffness; (c) the deck slab is 200 mm thick; and (d) each girder has a depth of 690 mm below the deck slab and a width of 460 mm. The bridges were analyzed under a centrally-located pair of vehicle axles, 1.22 m apart and each weighing 90 kN. Two transverse positions of the vehicles were considered: for one position the loads were placed eccentric and for the other, the load were placed centrally. The idealizations and the loads used in the pretest analysis are shown in Figure 10.





(b) Elevation of the simply-supported bridge



(c) Elevation of the equivalent bridge representing the fixed-ended structure

Figure 10/ The idealizations used for pre-test analyses

Table 1 lists results of analysis for the structure with simple supports, and Table 2 the results for the structure that is equivalent to the rigid frame bridge.

These tables also list the average girder moments, deflections at mid-span, and the distribution factors (DF) for both moments and deflections. It is recalled that DF for a girder is the ratio of the actual and average responses of the girder at the transverse section under consideration.

The results given in Tables 1 and 2 clearly show that for the idealizations under consideration, the maximum values of DF for deflections are within about 5% of the corresponding DF for girder moments. In the light of this observation, it was noted with confidence that for the Wyandotte Street Bridge, the measurement of girder deflections alone can give a fairly good idea of the load distribution characteristics of the bridge.

The DF for deflections for the eccentric load case are plotted in Figure 11 for both the idealizations. Similarly, the DF for the central loading case are plotted in Figure 12. It can be seen in these figures, that irrespective of the idealization, the transverse distribution profile of the bridge is fairly peaky with the immediately loaded girders sustaining most of the applied loading. This observation implies that the outer girders being well removed from the roadway by the raised sidewalk, are likely to experience only a very small fraction of the loads applied on the roadway. Consequently, it was concluded that the highly deteriorated outer girder on the southern side of the bridge, does not pose any threat to the vehicle sustaining capacity of the bridge.

The pretest analysis showed that for the rigid frame bridge subjected to only one vehicle, the most heavily loaded internal girder sustains about 70% of one line of wheels; this load acceptance transforms into a D value of 2.10 m for longitudinal moments. For loading with two side-by-side vehicles, the corresponding D value was found to be 1.52 m. It is interesting to note that for one-vehicle loading, four of the immediately loaded girders sustain about 95% of the total vehicle.

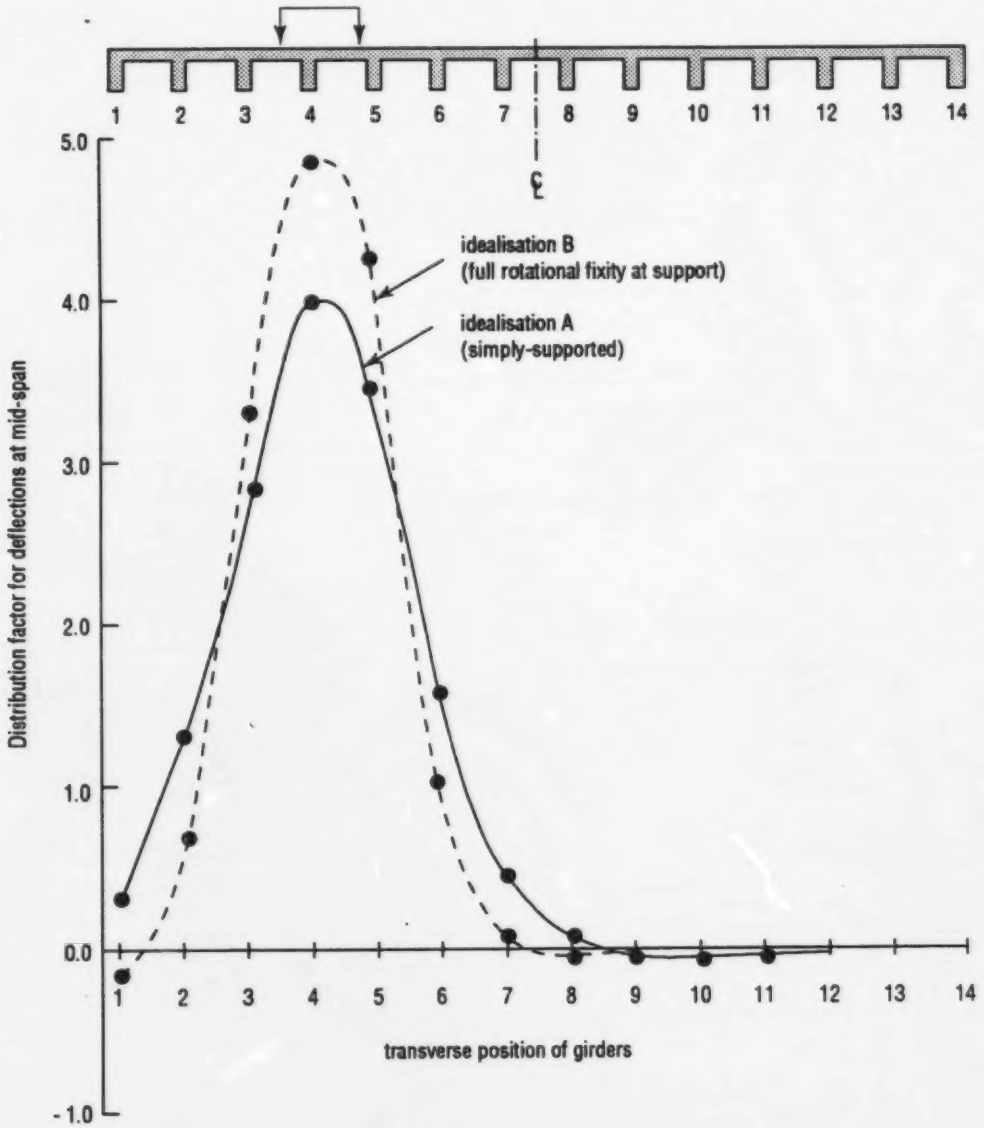


Figure 11/ Distribution factors for mid-span deflections for the eccentric load case

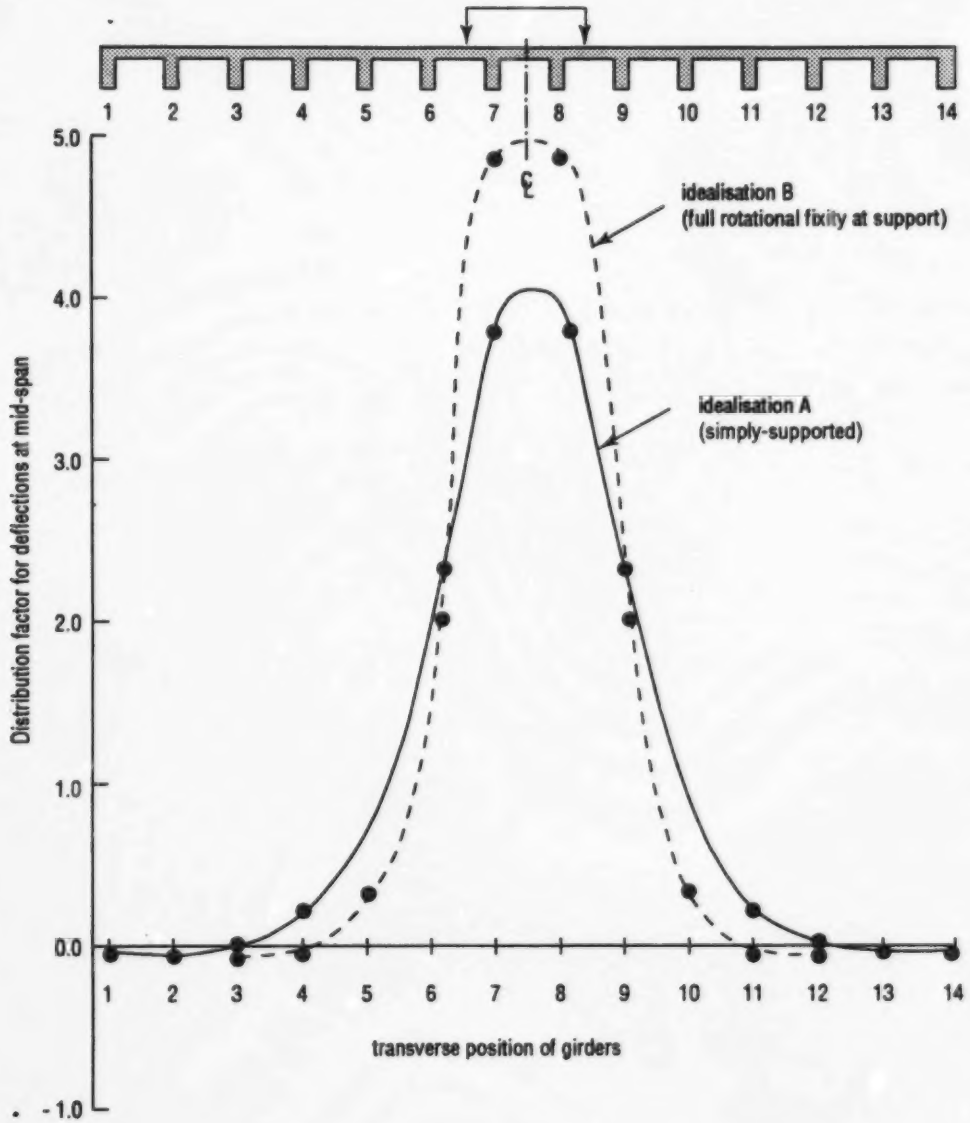


Figure 12/ Distribution factors for mid-span deflections for the central load case

5/ Details of Instrumentation and Testing

5.1/ Details of Instrumentation

As discussed in Chapter 4, it was decided initially to monitor the structure mainly through deflections of the girders at the mid-span. The removal of loose concrete from the girders, before the test, exposed enough bottom steel reinforcement and bottom flanges of the embedded steel girders, to enable the installation of electrical resistance strain gauges at the mid-span of seven of the middle eight girders; the eighth girder (10th from the south side) was found to have sound concrete even at the bottom. Accordingly, it was decided to install unidirectional resistance gauges to the exposed steel of the seven girders. It is noted that the electrical resistance gauges were applied to steel only after grinding the surface to a shining finish. A strain transducer, measuring average strains over a length of about 250 mm, was installed to measure longitudinal strains on the 10th girder from the southern end.

In order to assess the degree of rotational restraint at the end of the girders, the top and bottom longitudinal strains of the girders near their supports was measured. Strain transducers and, in one case, an electrical resistance gauge were used to measure these strains. Typical locations of strain transducers near the girder supports, are shown in Figure 14.

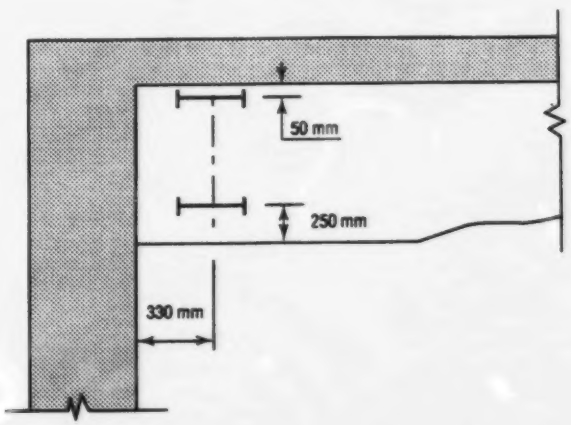


Figure 14/ Typical location of strain transducers near the girder supports

The identification numbers of deflection transducers installed at the midspan are shown in Figure 13(a). Similarly, the identification numbers of the strain measuring devices at the mid-span are shown in

Figure 13(b). The identification numbers for strain measuring devices applied near the supports are shown in Figures 13(c) and (d). The single case in which an electrical resistance strain gauge was applied to steel to measure girder strains near the supports, can be seen in Figure 5. This figure also shows the strain transducer applied near the top of the girders.

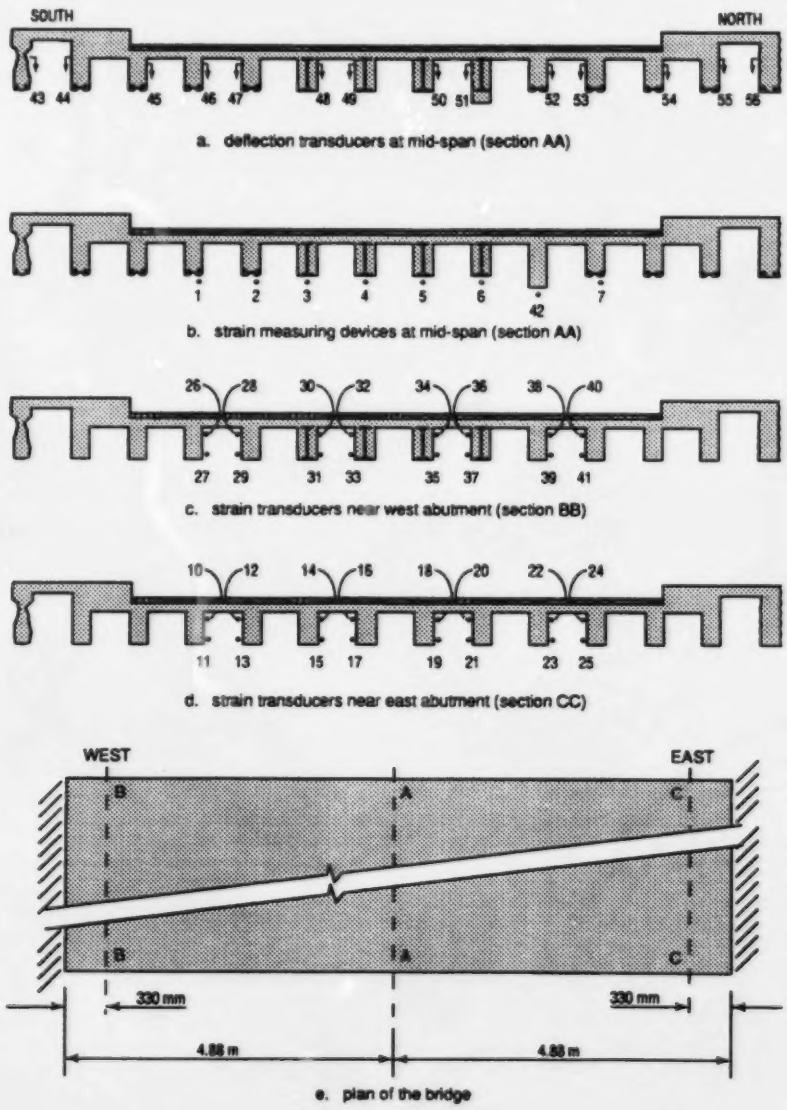

SOUTH
NORTH
43 44 45 46 47 48 49 50 51 52 53 54 55 56
a. deflection transducers at mid-span (section AA)
1 2 3 4 5 6 42 7
b. strain measuring devices at mid-span (section AA)
26 28 30 32 34 36 38 40
27 29 31 33 35 37 39 41
c. strain transducers near west abutment (section BB)
10 12 14 16 18 20 22 24
11 13 15 17 19 21 23 25
d. strain transducers near east abutment (section CC)
WEST
EAST
B
A
C
330 mm
330 mm
4.88 m
4.88 m
e. plan of the bridge

Figure 13/ Identification numbers for the instruments

A view of the bridge showing some of the instruments, is given in Figure 15. The two moveable scaffoldings used to install the instruments to the underside of this rather tall bridge, can also be seen in this view. Deflection transducers were attached to the sides of the girders where the concrete was generally more sound than at the bottom.

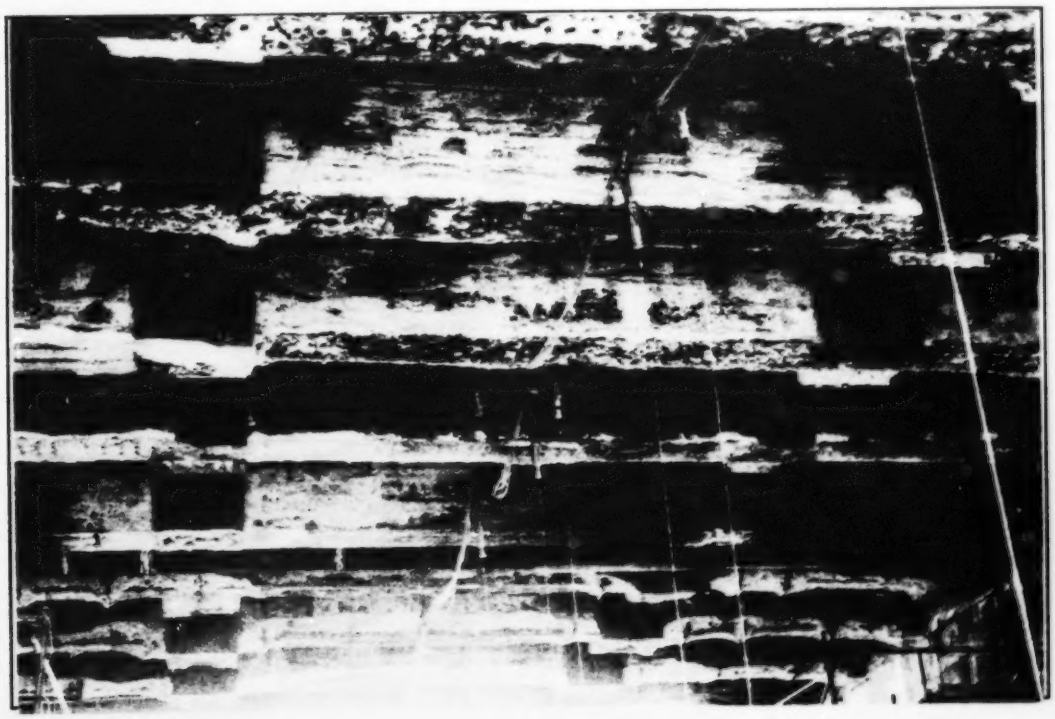


Figure 15/ A view of the bridge showing some of the instruments

5.2/ Test Loads

The two Ministry test vehicles were used to test the bridge. Axle spacings and other relevant dimensions of the test vehicles are shown in Figure 16. The bridge was tested mainly under the front dual axle tandem, i.e. axle Nos. 2 and 3, of the test vehicle. The weights of these axles were regulated by placing concrete blocks, each of which weighs about 9.33 kN, on the front portions of the test vehicles. The bridge was tested under four load levels including the empty test vehicles.

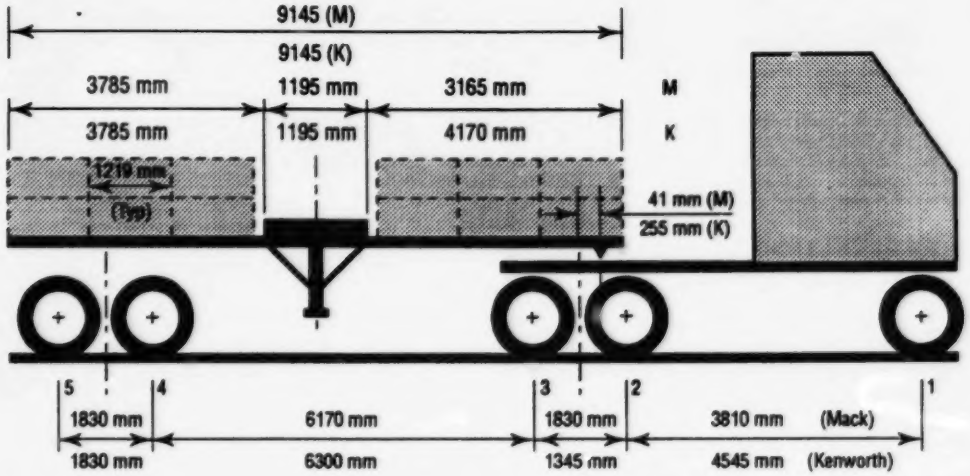


Figure 16/ Dimensions of the two test vehicles

Axle loads corresponding to the four load levels are given in Table 3 for the two test vehicles. As can be seen in this table, both axle Nos. 2 and 3 are shown to be carrying equal loads despite the fact that the pins transferring the loads to the front tandem are eccentric to the centres of the two-axle groups, as can be seen in Figure 16. This is done with the knowledge that the axle groups with unequal loads induces higher load effects in bridges than the group with equal loads. Consequently, the assumption of equal axle weights in an eccentrically loaded tandem leads to safe-side errors in the process of bridge evaluation by proof testing. This aspect is explained further in Chapter 7.

The decision to subject the bridge posted for 5 tonnes, to even an empty test vehicle, which weighs considerably more than 5 tonnes, was based entirely on engineering judgement. Only when the measured responses confirmed that the empty trucks did not cause any distress to the bridge was a decision taken to gradually increase the test loads. The decision to stop the test loading at level 4 was not caused by any sign of distress or impending stress in the bridge; instead, it was caused from concerns of safety regarding the vehicle, the axle loads on the front tandem of which were already very heavy.

The bridge was tested with a single test vehicle in each of five transverse positions, and with two side-by-side vehicles in two eccentric transverse positions. These transverse positions, also referred to as transverse lines, are shown in Figure 17 with the identification numbers. Corresponding to each transverse line, the vehicle or vehicles were stopped at seven longitudinal positions along the span.

These longitudinal positions, also referred to as longitudinal positions, are shown in Figure 18 with the identification numbers. The vehicle, or vehicles, were brought onto the bridge at crawling speed, and the readings of the instruments were recorded a few seconds after the vehicle, or vehicles, had been brought to a halt at the prescribed position on the bridge. The readings of the instruments were also recorded immediately before and after the vehicles were brought on the bridge.

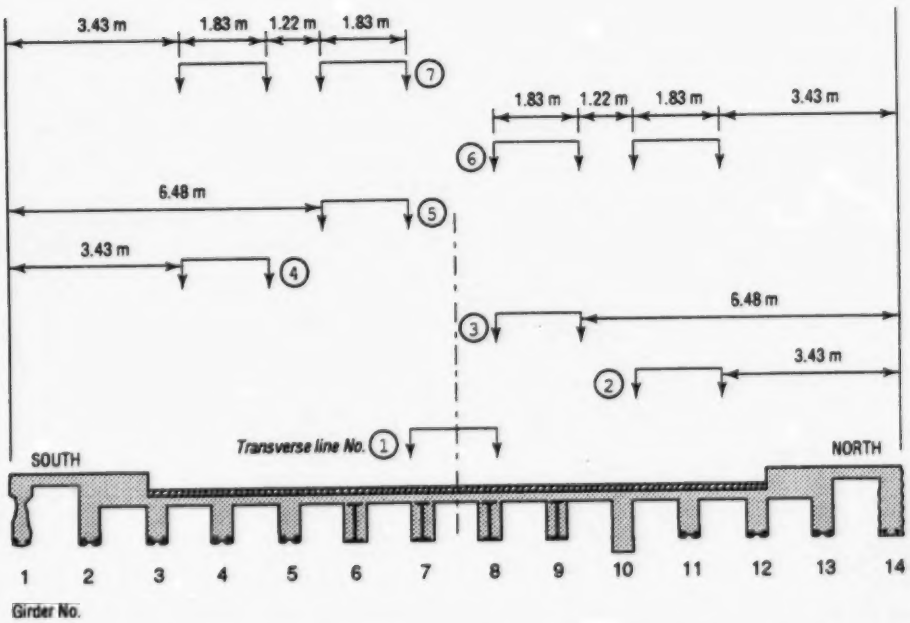


Figure 17/ Transverse positions of test vehicles and their identifying numbers

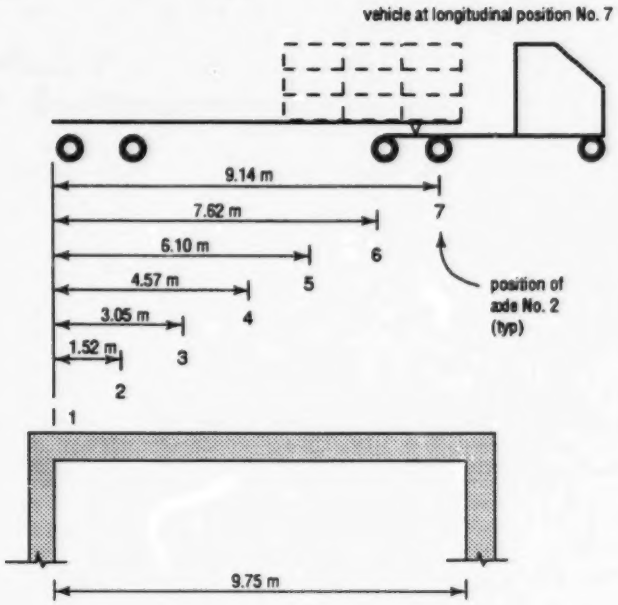


Figure 18/ Longitudinal positions of test vehicles and their identifying numbers

6/ Recorded Data

Girder strains, at mid-span, due to various longitudinal positions and load levels of the test vehicles are plotted in Figure 19. With the help of the plots of Figure 19, it can be readily shown that the responses due to vehicles at longitudinal position No. 5 are, at most, 4% short of the corresponding maximum responses. The data corresponding to longitudinal position 5, are recorded in this report with knowledge that these data are likely to be smaller than the actual maximum responses by only a very small margin.

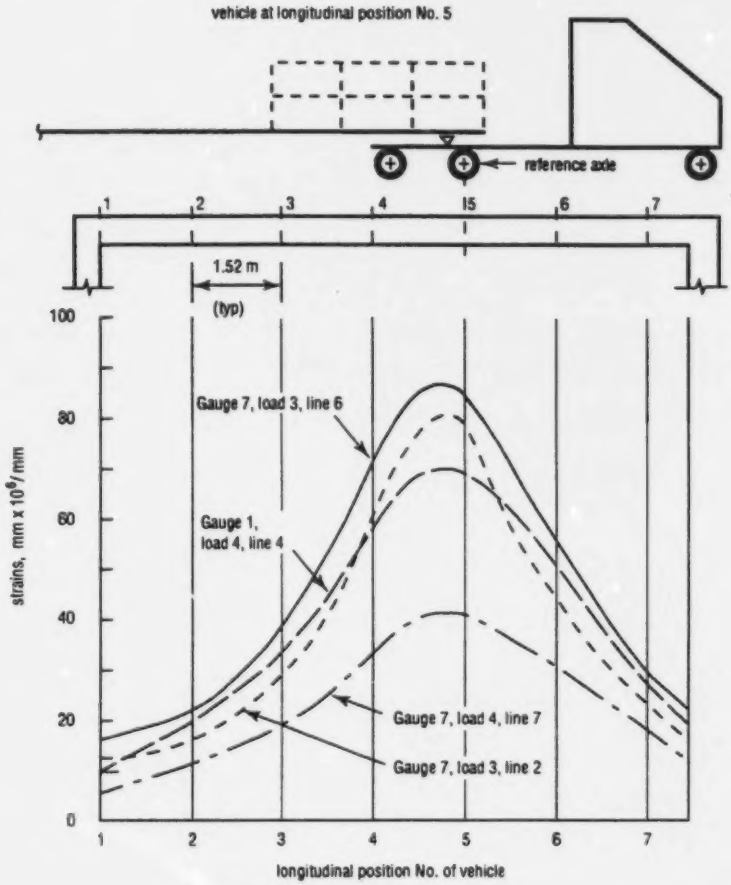


Figure 19/ Strains plotted against longitudinal positions of the vehicle

For loading with only one vehicle, the bridge was tested with the test load at four levels; for these load cases, the data are recorded for the 4th, i.e. the highest, load level. Testing with two side-by-side vehicles was performed with the test loads at three load levels; accordingly, for these load cases, the data are recorded for only load level 3, i.e. the highest, load level.

Mid-span deflections of girders are given in Table 4. It can be seen in this table that the magnitudes of deflections are very small being less than 1 mm in most cases. As discussed later, the accuracy of the deflection measuring system is such that complete reliance can not be placed on the magnitudes of measured deflections; they should be regarded as providing only a qualitative measure of the actual deflections. Strains of the steel in the girders, on the other hand, were measured accurately and could provide a direct measure of the strength of the girders. These strains are given in Table 5.

Longitudinal strains of girder concrete near the supports, are listed in Table 6. These strains because of being rather small in magnitude, appear to be inconsistent. They do not yield immediately useful information other than an indication of the presence of partial rotational restraint at the supports.

6.1/ Checking of the Data

One of the ways to establish whether the instruments are working properly, is to perform a summation check on the measured responses. In such a check, the sums of responses due to two separate load cases are compared with the corresponding responses due to a load case comprising these two individual load cases. As can be seen in Figure 17, vehicles in transverse position No. 6 are equivalent to the sum of vehicles in transverse position Nos. 2 and 3. Similarly, vehicles in transverse position No. 7 are the result of superposition of vehicles in transverse position Nos. 4 and 5.

It was found that for all cases, the summation checks confirmed that the instruments were working properly. Two examples of summation checks involving the vehicles at load level 3 and in longitudinal position No. 5, are provided in Table 7. It can be seen in this table that the responses due to a two-vehicle load case are, indeed, very close to the corresponding sum of responses for the one-vehicle load cases.

6.2/ Load-Response Relationship

Some of the larger strains and deflections measured during the test are listed in Tables 8 and 9, respectively, for the different load levels. It was observed from these and other responses that both the strains and deflections varied approximately linearly up to load level 3. Thereafter, the structure seems to stiffen up indicating that certain components begin to act compositely at higher load levels.

Table 8 also lists the residual strains which are the differences between the strain measurements before and after the placing of the test loads on the bridge. Small magnitudes of residual strains observed in every test; they are caused by the "drifting" of the instruments and by the response of the structure to temperature changes that might have occurred during the time for which the test loads were on the bridge. Only when residual strains are large in magnitude, say greater than about 5%, can it be inferred that the structure acted in a non-elastic manner. It can be seen that the residual strains given in Table 8 are relatively small indicating that the structure behaved in an elastic manner even up to load level 4.

The residual deflections given in Table 9 might appear excessively large compared to the measured deflections. The residual deflections of such magnitude are typical of the drift encountered in deflection measuring devices; consequently, they should not be regarded as indicative of non-elastic response especially when strain measurements have already confirmed elastic response.

The load-strain curves for certain cases are plotted in Figure 20. It can be seen in this figure that all the curves follow nearly a straight line up to load level 3. The curling up of these curves towards the load-axis after level 3 indicates the stiffening action discussed above. The same stiffening action can also be observed in the load-deflection curves plotted in Figure 21. It is noted that because of the small magnitudes of measured deflections, these plots are not as reliable as the load-strain plots.

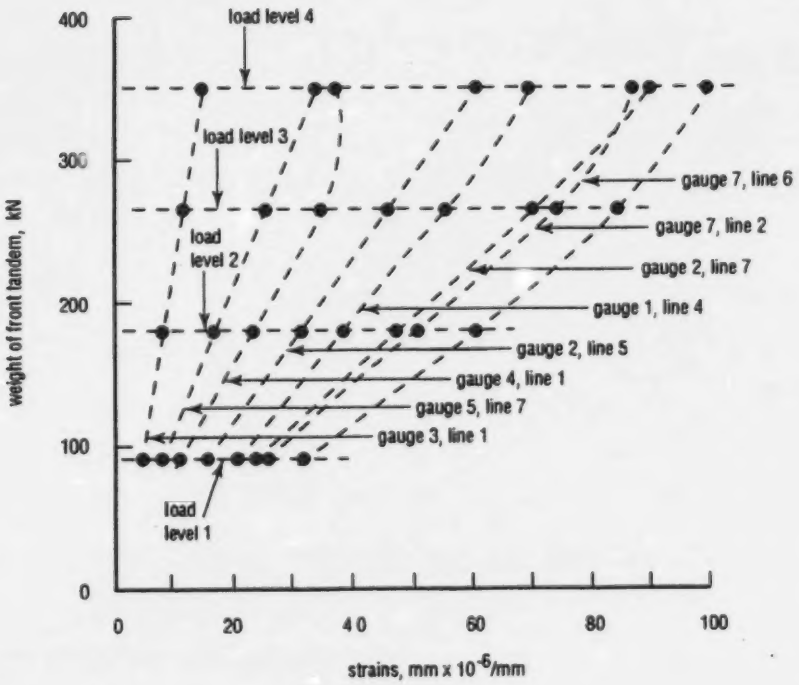


Figure 20/ Plots of strains vs. load corresponding to longitudinal position No. 5

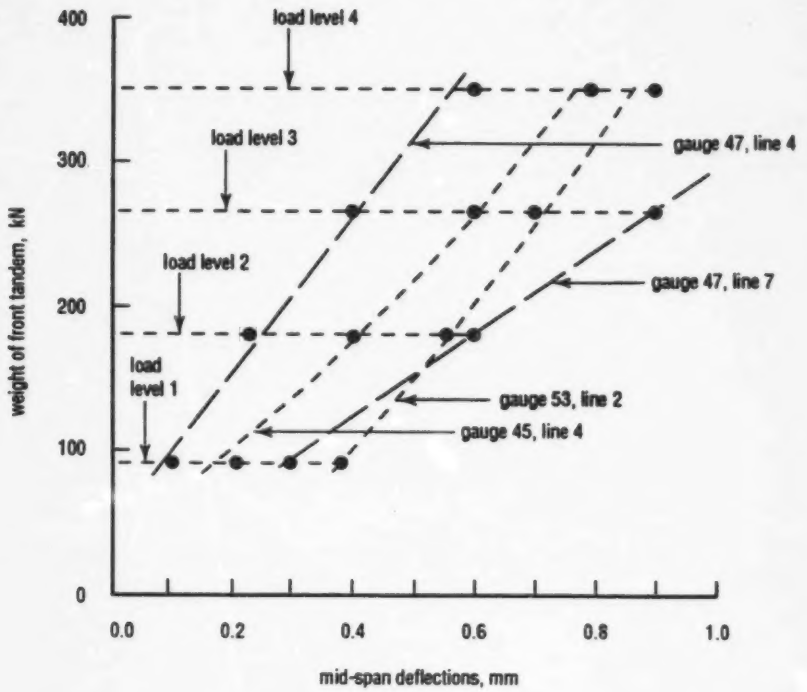


Figure 21/ Plots of deflections vs. loads corresponding to longitudinal position No. 5

The stiffening action of the structure at higher load levels is encouraging because it signifies that the structure can sustain higher loads than predicted by linear extrapolation of measured responses.

6.3/ Rotational Restraint at Supports

Girder strains near the supports, are shown in Table 6. It can be seen that the strain values recorded in this table are rather small in magnitude and somewhat erratic. This behaviour may be due to the delamination of the concrete cover over the reinforcement, as was observed in the middle regions of the concrete. It is possible that the concrete cover near the girder support to which the strain transducers were attached may not be as sound as it appeared. The strains given in Table 6 do, however, point towards the presence of compressive strains near the bottom of the girders. This observation confirms that the girders have some rotational restraint at their ends.

A quantitative assessment of the degree of rotational restraint at the girder supports is provided by Figure 22 in which measured deflections corresponding to load level 4 are compared with deflections

given by the computer program SECAN1 for the two idealizations which are discussed in Chapter 4 and illustrated in Figure 10. To facilitate the comparison, the deflections due to two load cases, the transverse positions of which are mirror images of each other, are drawn on the same side of the cross-section by using two girder numbering systems as shown in the figure; further, the measured deflections are adjusted slightly to within the accuracy of the measuring system, to enable the drawing of smooth deflection profiles.

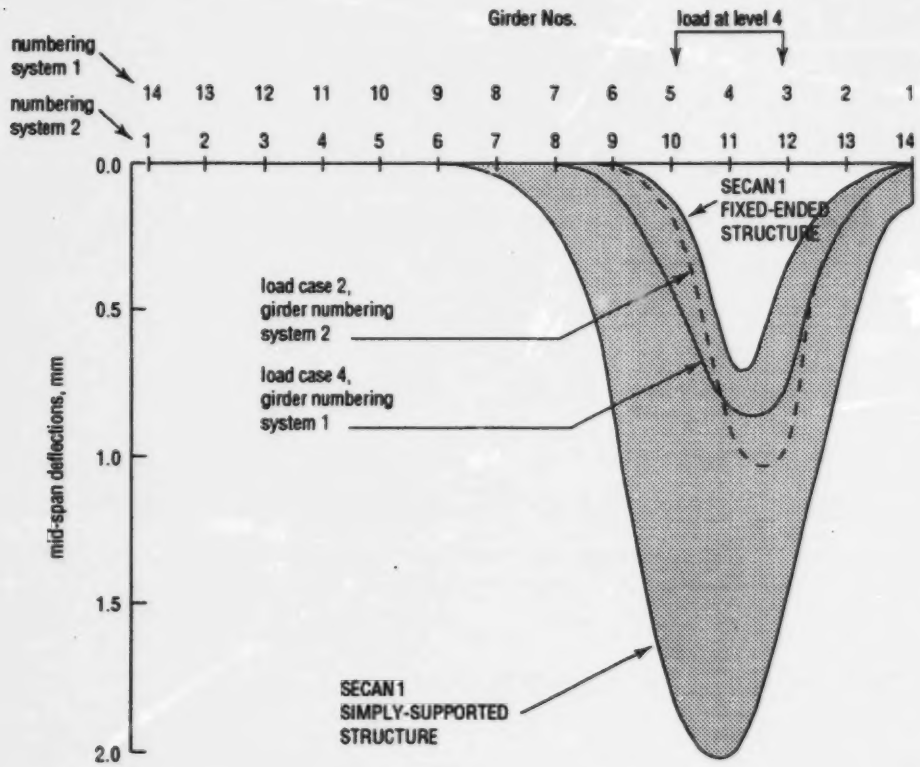

Girder Nos.
load at level 4
numbering system 1
numbering system 2
14 13 12 11 10 9 8 7 6 5 4 3 2 1
1 2 3 4 5 6 7 8 9 10 11 12 13 14
0.0
0.5
1.0
1.5
2.0
mid-span deflections, mm
SECAN1 FIXED-ENDED STRUCTURE
load case 2, girder numbering system 2
load case 4, girder numbering system 1
SECAN1 SIMPLY-SUPPORTED STRUCTURE

Figure 22/ Comparison of measured deflections at the mid-span with analytical deflections

It can be seen in Figure 22 that the measured deflections are bracketed entirely by the analytical deflections corresponding to the fixed-ended and simply supported cases. The measured deflections are closer to the fixed-ended case than simply-supported case, confirming that the girders have very nearly full rotational restraint at their supports.

Two differences between the actual structure and the idealizations used in the analysis should be noted: (a) The girder depth below the deck slab was taken in the idealization as 690 mm corresponding to the actual dimension of about 610 mm of the sound concrete left in the girders after removal of loose concrete; (b) the idealization did not take account of the fact that the outer two girders were somewhat deeper than the others, as can be seen in Figure 1. Despite these minor differences in the actual and idealized structures, the analysis was able to predict fairly accurately the pattern of transverse distributions mid-span deflections, as can be seen in Figure 22.

7/ Strength Evaluation

From Table 5 it can be seen that the maximum strain in the bottom steel of the girders due to test loads was only 88 με, which corresponds to a stress of approximately 18 MPa. This stress level is very low compared to the likely minimum yield stress of about 200 MPa. The maximum strain corresponding to the two-vehicle loading at level 4, obtained by super-position of the strains due to single-vehicle loadings, is about 101 με. This strain is equivalent to about 20 MPa.

If it is assumed very conservatively that the dead loads exert twice as much stresses in the bottom steel of the girders as the applied proof loading, than the dead load stress is estimated to be only 40 MPa leaving approximately 160 MPa for live and other loads. Since the stress due to the proof load is only one-eighth of this estimated capacity and since the structure behaved in an elastic manner under the proof load, there should be no hesitation in accepting the proof load as a very conservative estimate of the live load carrying capacity of the bridge. A careful consideration of the likely failure mode of the bridge, leads to the conclusion that the failure will be governed by tensile stresses in the bottom reinforcement of the girders.

7.1/ Equivalence with OHBDC Evaluation Loads

For analytical evaluation of the load carrying capacity of a bridge, the Ontario Highway Bridge Design Code (OHBDC, 1983) has provided an evaluation loading which comprises a five-axle truck, the axle weights and dimensions of which are shown in Figure 23. It can be shown readily that the 1.2 m apart dual axle group of this truck governs the load effects in the 9.8 m span Wyandotte bridge. If the two-axle groups of the test vehicle also had axles spaced at 1.2 m, the equivalence of the test loading

with the evaluation loading would have been straightforward for the bridge under consideration. In the absence of such facility, the equivalence could have been sought on the basis of the maximum moments induced by the two sets of loadings. However, even this device is not easily applicable to the Wyandotte bridge because of uncertainty in the degree of rotational restraint that this bridge has at its supports.

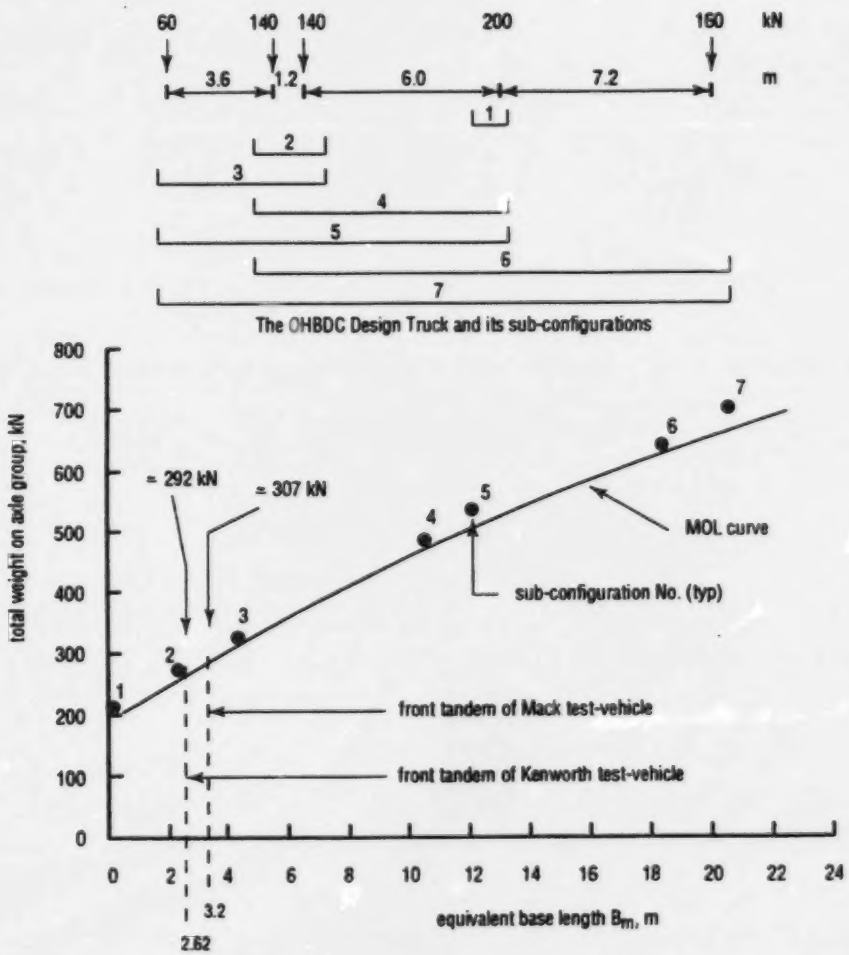


Figure 23/ Total axle weights plotted against equivalent base length

The equivalence between the evaluation and test loadings can be readily achieved, by using the information documented in the commentary to the OHBDC (1983).

It has been shown in several references, including the commentary to the OHBDC (1983), that a set of N concentrated loads $P_1, P_2, ..., P_N$, can be represented by an equivalent uniformly distributed load (UDL) in such a way that the bending moments and shear forces induced by the two sets of loads in beams of all spans are nearly the same. The weight of the UDL, $W = \sum_{i=1}^{N} P_i$, and the length B_m, referred to as the equivalent base length, is given by

$$B_m = \frac{4}{W} \sum_{i=1}^{N} \left| P_i X_i \right| - \frac{2(N-1)}{6NW^2} \sum_{i=1}^{N} (P_i X_i) \qquad (7.1)$$

where X_i is the distance of the concentrated loads from the load that is closest to the centre of gravity of the group. It can be shown that for two equal loads, B_m = 1.75 x the distance between the loads.

The OHBDC truck loading for design and evaluation loads represents closely, although not exactly, the maximum observed load (MOL), W_{MOL}, which is given by the following equation

$$W_{MOL} = 9.806 (20.0 + 3.0B_m - 0.0325B_m) \qquad (7.2)$$

where W_{MOL} is in kN and B_m in metres.

The MOL curve obtained by the above equation is plotted in Figure 23. This figure also shows the weights of the various sub-configurations of the OHBDC truck. It can be seen that the weights of the OHBDC trucks are slightly above the MOL line. In particular, the weight of the two axle group having B_m of 2.1 m is about 9% above MOL and the three-axle group identified as configuration No. 2 in Figure 23, is about 5% above MOL.

The equivalent base lengths for the dual axles of the OHBDC truck and the two test vehicles are shown in Figure 24, with those for the latter being 2.62 m and 3.20 m, respectively. By assuming the design loads to be 8% and 7% above the MOL for these two equivalent base lengths respectively, one gets an equivalent evaluation load of 292 kN for the dual axle of the Kenworth test vehicle and 307 kN for the Mack test vehicle. The implication of these equivalent loads is that the test loading can be considered equivalent to the evaluation load only if it carries these equivalent loads.

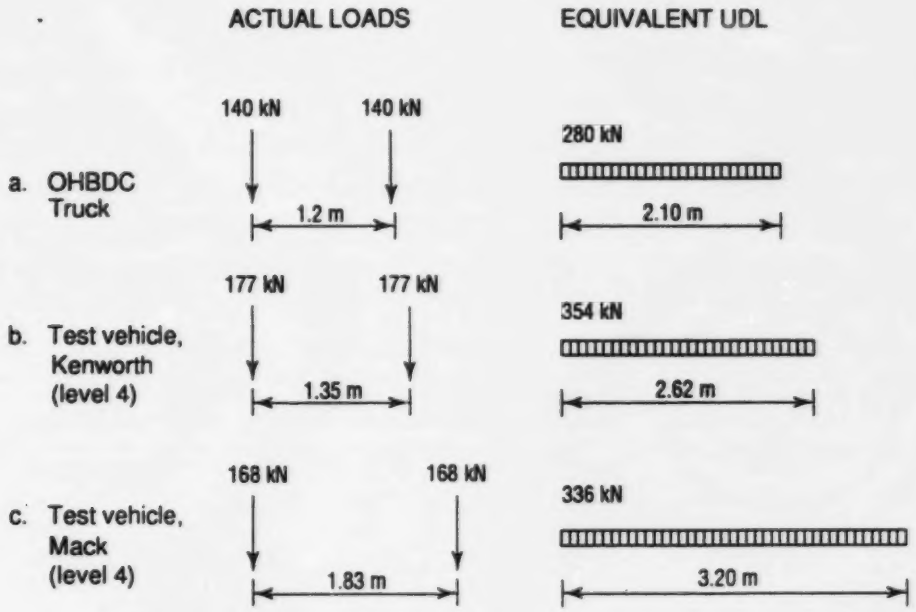


Figure 24/ Equivalence of concentrated loads with uniformly distributed loads

The above calculations for equivalence with evaluation loads are done by assuming that the two axles of the test vehicles are loaded evenly. As can be seen in Figure 16, the axles of the front tandem of both the test vehicles are loaded eccentrically because of which the loads are distributed unevenly between the axles. It can be shown that the equivalent base length of a unevenly loaded dual axle group is smaller than that of an evenly loaded group. The consequence of assuming even distribution of load among the axles is, hence, an overestimation of the equivalent evaluation load. This overestimation, in term, makes the ensuing calculations a little more conservative i.e. safer.

7.2/ Scale-down Factor

The OHBDC (1983) defines through Clause 14-7.2.1 a quantity F which is called the scale-down factor and is given by

$$F = \phi R_n - \frac{\text{load effects due to factored dead loads}}{\text{load effects due to factored live loads}} \qquad (7.3)$$

The numerator of the above equation represents the strength of the bridge that is available for live loads. For evaluation by proof testing, it can be demonstrated that the proof load reduced appropriately to account for the dynamic load allowance, represents the strength of the bridge that is available for live loads. In this case, the denominator of the above equation should accordingly be the equivalent evaluation load corresponding to the test load configurations as discussed in Section 7.1, multiplied by the load factor and the modification factor for multi-presence of vehicles.

The dynamic load allowance, I, for bridges with span less than 22 m is specified to be 0.30 by the OHBDC (1983); the modification factor, mf, for two-lane loading is specified to be 0.90 and the live load factor to be 1.25 if the bridge is to be re-evaluated in five years.

For I = 0.30, the equivalent static proof loads on the front tandems of the Kenworth and Mack vehicles are 354/1.3 (= 272 KN) and 336/1.3 (= 258 kN), respectively.

Accounting for m_f = 0.9, the evaluation loads equivalent to the front tandems of the Kenworth and Mack vehicles are 0.9 x 292 (=263kN) and 0.9 x 307(=276 kN), respectively. Hence, the scale-down factor F is given by

$$F = \frac{272 + 258}{1.25(263 + 276)} = 0.79$$

It is emphasized that the two-lane proof loading of the structure despite it having four design lanes, is considered adequate because as pointed by Bakht and Jaeger (1985) the governing load case for slab-on-girder bridges with two or more lanes is one with only two side-by-side vehicles.

7.3/ Calculation of Posting Loads

For F = 0.79, the charts given in Figures 14-7.3(a), (b) and (c) of the OHBDC (1983) give the posting loads of 19, 26 and 35 tonnes respectively for level 1, 2 and 3 evaluation loads. This leads to a single posting load of 19 tonnes and a triple posting of 19, 26 and 35 tonnes.

8/ Recommendations

Based on a close examination of the data collected during the test, it is recommended that:

a) The restriction of limiting the bridge to only two lanes of traffic can be lifted safely and all the four lanes can be opened to traffic.

b) The current posted weight limit of 5 tonnes can be upgraded to either a single posting of 19 tonnes or to a triple posting of 19, 26 and 35 tonnes.

It is emphasized that the bridge has ample additional capacity beyond that reflected by the proof loads and, therefore, the recommended measures are quite safe.

Since the outer girder on the southern side of the bridge is highly deteriorated, the southern sidewalk on the bridge must remain closed until the girder is replaced by an adequately strong girder.

References

[1] Bakht, B. and Jaeger, L.G. 1985. Bridge Analysis Simplified. McGraw Hill, N.Y.

[2] Jaeger, L.G. and Bakht, B. 1989. Bridge Analysis by Microcomputer. McGraw Hill, N.Y.

[3] Ontario Highway Bridge Code. 1983. Ministry of Transportation of Ontario, Downsview, Ontario.

[4] Commentary to Ontario Highway Bridge Design Code. 1983. Ministry of Transportation of Ontario, Downsview, Ontario.

Table 1/ Results of analysis by SECAN1 of the simply supported bridge

load case	girder No.	mid-span moments			mid-span deflections		
		avg. mt. / girder, kN•m	girder moment, kN•m	DF *	avg. girder deflections, mm	girder deflections, mm	DF *
eccentric	1	27.31	7.51	0.28	0.259	0.079	0.30
	2		30.24	1.11		0.330	1.27
	3		81.83	3.00		0.762	2.93
	4		112.28	4.11		1.041	4.00
	5		98.43	3.61		0.889	3.41
	6		38.46	1.41		0.406	1.56
	7		11.94	0.44		0.127	0.49
	8		2.18	0.08		0.023	0.09
	9		- 0.20	- 0.01		- 0.002	- 0.01
	10		- 0.39	- 0.01		- 0.004	- 0.02
	11		- 0.19	- 0.01		- 0.002	- 0.00
	12		- 0.07	- 0.00		- 0.001	- 0.00
	13		- 0.00	- 0.00		- 0.000	- 0.00
	14		- 0.00	- 0.00		- 0.000	- 0.00
central	1	27.31	- 0.34	- 0.01	0.259	- 0.004	- 0.01
	2		- 0.44	- 0.02		- 0.005	- 0.02
	3		0.22	0.01		0.002	0.01
	4		4.45	0.16		0.048	0.19
	5		19.00	0.70		0.203	0.78
	6		55.12	2.20		0.558	2.15
	7		112.67	4.13		1.01	3.90

** DF (distribution factor) = actual response / average response*

Table 2/ Results of analysis by SECAN1 of the bridge with rotationally fixed supports

load case	girder No.	mid-span moments			mid-span deflections		
		avg. mt. / girder, kN•m	girder moment, kN•m	DF *	avg. girder deflections, mm	girder deflections, mm	DF *
eccentric	1	16.91	- 0.16	- 0.01	0.072	- 0.001	- 0.01
	2		10.06	0.60		0.048	0.67
	3		54.82	3.24		0.236	3.27
	4		83.26	4.92		0.356	4.92
	5		71.97	4.25		0.305	4.92
	6		16.04	0.94		0.076	1.06
	7		1.24	0.07		0.006	0.08
	8		- 0.39	- 0.02		- 0.002	- 0.03
	9		- 0.15	- 0.00		- 0.001	- 0.01
	10		- 0.02	- 0.00		- 0.000	- 0.00
	11		0.00	0.00		0.000	0.00
	12		0.00	0.00		0.000	0.00
	13		0.00	0.00		0.000	0.00
	14		0.00	0.00		0.000	0.00
central	1	16.91	0.00	0.00	0.072	0.000	0.00
	2		- 0.05	- 0.00		- 0.000	- 0.00
	3		- 0.24	- 0.01		- 0.001	- 0.02
	4		- 0.28	- 0.02		- 0.001	- 0.02
	5		3.82	0.23		0.018	0.25
	6		29.35	1.74		0.135	1.86
	7		85.74	5.07		0.356	4.92

** DF (distribution factor) = actual response / average response*

Table 3/ Axle weight of the two test vehicles

Test vehicle	load level No.	No. of conc. blocks	weight in kN for axle No.					Total weight, kN
			1	2	3	4	5	
Kenworth	1	0	44	46	46	41	41	218
	2	12	48	90	90	51	51	330
	3	24	52	133	133	62	62	442
	4	36	57	177	177	73	73	557
Mack	1	0	41	44	44	39	39	207
	2	12	49	85	85	50	50	319
	3	24	57	127	127	61	61	433
	4	36	65	168	168	72	72	545

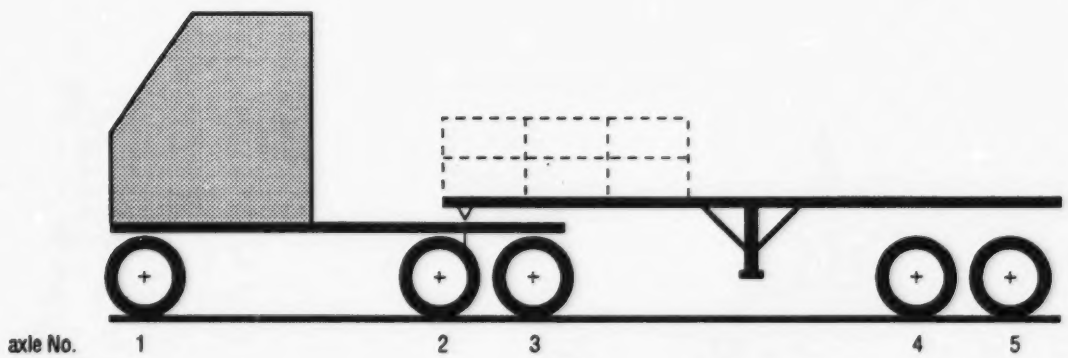
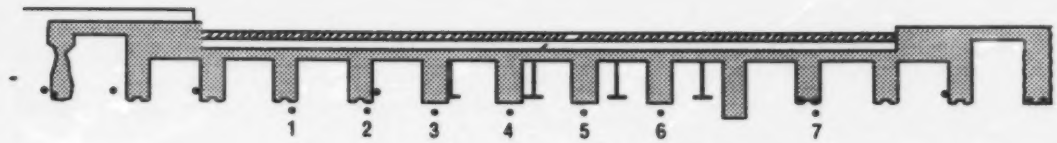


Table 4/ Girder deflections at mid-span due to vehicle/s at levels 3 and 4 and at longitudinal position No. 5

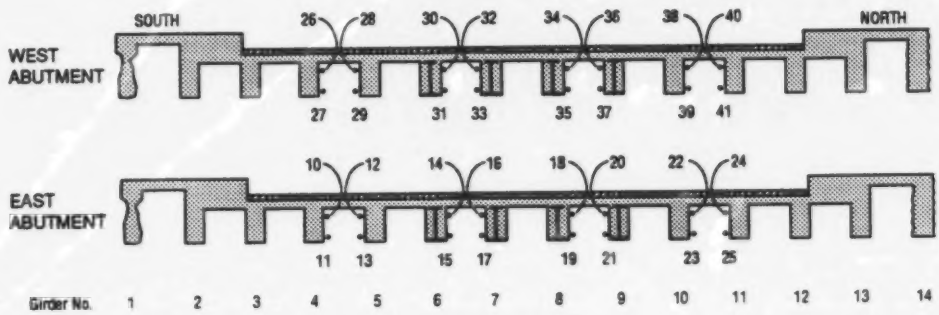
load level	trans. line No.	deflections in mm at gauge No.													
		43	44	45	46	47	48	49	50	51	52	53	54	55	56
4	1	0.0	0.1	0.0	0.0	0.0	0.2	0.4	0.3	0.0	0.0	0.0	0.0	0.0	- 0.1
	2	0.0	0.0	0.0	0.0	- 0.1	0.0	- 0.1	- 0.2	- 0.1	0.2	0.9	1.0	0.1	0.1
	3	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.7	1.0	0.8	- 0.2	- 0.2	- 0.2	- 0.2
	4	0.0	0.0	0.8	0.6	0.6	0.1	0.1	- 0.1	- 0.3	- 0.2	0.0	0.1	0.1	0.1
	5	- 0.1	0.0	0.0	0.2	0.7	0.6	0.3	0.1	- 0.1	- 0.2	0.2	0.2	0.2	0.2
3	6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.3	0.5	0.8	0.7	0.7	0.0	0.0
	7	0.1	0.0	0.8	0.8	0.9	0.5	0.4	0.2	0.4	0.3	- 0.2	- 0.3	- 0.2	- 0.2

Table 5/ Girder strains at bottom flange at mid-span due to vehicle/s at load levels 3 and 4 and at longitudinal position No. 5



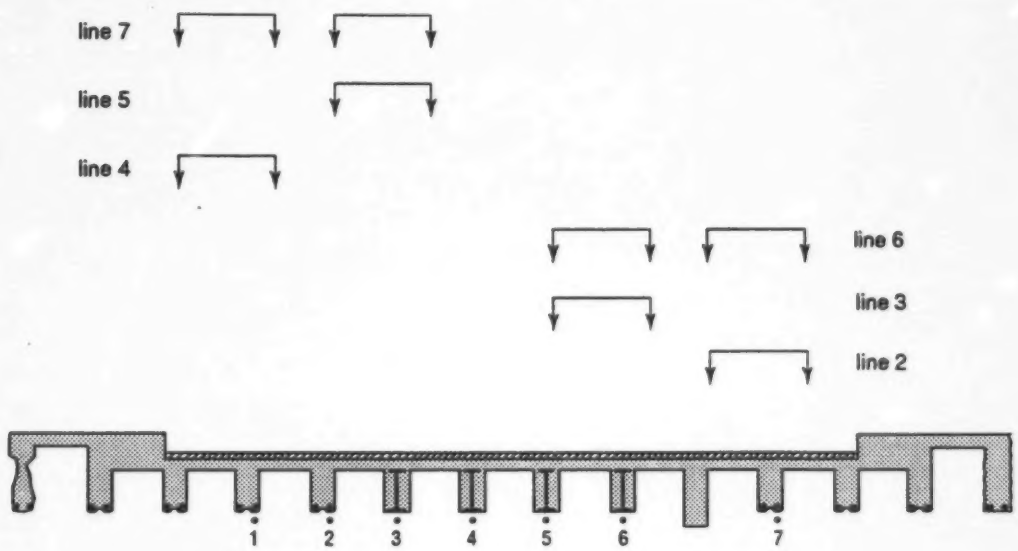
load level	trans. line No.	strains, mm x10^6/ mm at gauge No.						
		1	2	3	4	5	6	7
4	1	-1	3	14	37	35	10	-2
	2	-1	-1	-1	-1	0	5	88
	3	-1	-1	0	8	33	30	13
	4	68	29	6	0	-1	-1	-1
	5	16	62	40	25	6	0	-1
3	6	-1	-1	0	6	25	27	83
	7	68	71	35	20	5	0	1

Table 6/ Longitudinal girder strains near supports due to vehicle/s at load levels 3 and 4 and longitudinal position No. 5



top/ bottom	load level No.	trans. line No.	west/ east	strains, mm x 10^6/ m, in girder No.							
				4	5	6	7	8	9	10	11
top	4	1	W	0	2	-3	5	5	-2	-1	-3
			E	-3	-1	-1	-3	-3	0	-3	0
		2	W	2	0	-3	-1	0	2	3	10
			E	-4	-2	1	-1	-2	-1	1	7
		3	W	0	1	0	2	1	-7	2	0
			E	1	0	-4	-1	0	-7	3	0
		4	W	3	0	0	-2	-4	2	-1	1
			E	2	-1	1	0	-1	1	1	0
		5	W	1	5	-6	0	1	0	0	1
			E	3	0	0	3	0	2	-1	0
	3	6	W	-3	-1	-1	0	4	-6	6	8
			E	-2	-1	-2	-2	1	-2	2	3
		7	W	-2	4	-4	1	-1	-1	0	0
			E	8	2	-2	-1	0	0	-1	0
bottom	4	1	W	2	0	2	-8	-9	-4	-2	0
			E	0	1	1	-4	-5	-9	-2	-1
		2	W	3	-2	-1	0	0	0	-3	-15
			E	-3	1	-1	1	1	-1	-8	-6
		3	W	1	0	4	0	0	-4	-7	-2
			E	3	1	-1	3	-1	-2	-3	-2
		4	W	-10	-13	-3	0	0	-1	-1	0
			E	-21	-8	-1	-1	0	-1	3	0
		5	W	2	-2	-3	-6	0	1	0	2
			E	-1	-5	-2	-6	1	-3	1	0
	3	6	W	1	2	1	-1	0	-5	-5	-12
			E	-1	1	-1	1	-2	-2	-9	-12
		7	W	-4	-17	-6	-9	-2	-1	-1	0
			E	-13	-23	-4	-8	-1	-2	-1	0

Table 7/ Summation checks corresponding to load level 3 and longitudinal vehicle position No. 5



trans. line No.	strains, mm x 10^6/ mm at gauge No.						
	1	2	3	4	5	6	7
2	0	1	0	0	0	5	76
3	0	0	1	6	24	23	9
Σ (2+3)	0	1	1	6	24	28	85
6	-1	-1	0	6	25	27	83
4	55	24	4	0	1	-1	-1
5	12	47	31	19	5	0	0
Σ (4+5)	67	71	35	19	6	-1	-1
7	68	71	35	20	5	0	1

Table 8/ Some of the larger strains corresponding to longitudinal position No. 5

transverse line No.	gauge No.	strains in mm x 10^6/ mm for load at level No.				"residual" strains, (mm x 10^6/ mm)	
		1	2	3	4	level 3	level 4
1	3	5	8	12	14	- 0.6	- 2.9
	4	10	23	34	37	- 0.6	- 4.9
	5	11	24	34	35	- 0.3	- 4.6
2	7	25	49	76	88	9.7	1.1
3	5	8	16	24	33	0.3	1.1
	6	8	17	23	31	0.6	1.1
4	1	18	37	55	68	0.0	0.3
	2	6	16	24	29	0.6	- 0.6
5	2	15	31	47	62	1.1	0.9
	3	11	21	31	40	0.6	- 0.6
	4	8	14	19	25	- 0.6	0.0
6	6	9	19	27	35*	1.4	
	7	29	58	83	100*	2.9	
7	1	24	47	68	84*		
	2	23	46	71	91*		
	3	13	24	34	46*		

** obtained by adding strains due to single vehicles*

Table 9/ Some of the larger deflections corresponding to longitudinal position No. 5

transverse line No.	gauge No.	deflections in mm for load at level No.				"residual" strains, deflections, (mm)	
		1	2	3	4	level 3	level 4
1	48	0.1	0.1	0.2	0.2	- 0.1	- 0.1
	49	0.1	0.2	0.6	0.4	0.1	- 0.1
	50	0.1	0.2	0.3	0.0	0.1	- 0.1
2	53	0.4	0.6	0.7	0.9	0.0	0.2
	54	0.4	0.6	0.7	1.0	- 0.1	0.2
3	50	0.0	0.1	0.3	0.7	0.0	0.2
	51	0.0	0.2	0.4	1.0	0.1	0.3
	52	0.0	0.2	0.3	0.8	- 0.1	0.2
4	45	0.1	0.4	0.6	0.8	0.1	0.1
	46	0.1	0.3	0.5	0.6	0.1	0.1
	47	0.1	0.2	0.4	0.6	0.0	0.1
5	47	0.3	0.3	0.5	0.7	0.1	0.1
	48	0.2	0.3	0.4	0.6	0.0	0.0
	49	0.1	0.1	0.3	0.3	0.0	0.1
6	51	0.3	0.3	0.5		0.2	
	52	0.4	0.4	0.8		0.1	
	53	0.2	0.6	0.7		- 0.2	
	54	0.2	0.6	0.7		- 0.1	
7	45	0.2	0.6	0.8		0.2	
	46	0.4	0.5	0.8		0.1	
	47	0.3	0.6	0.9		0.1	
	48	0.3	0.4	0.5		0.0	